Arc Welded Steel Building at West Philadelphia Works of the General Electric Company*

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THE invention of new methods of articulating steel truss members has invariably resulted in changes in the shapes of the members themselves. During the decade ending about 1890, iron and steel truss members were connected together largely by pins, each joint of the truss having one cylindrical pin; which, at first, was claimed to be so perfect that practically no secondary stresses occurred in the main members. American engineers argued against fate in favor of pin joints vs. riveted joints, but by 1890 it was recognized that friction between pin and truss members was so great that secondary stresses did exist. Moreover, riveted trusses were stiffer. So riveted work found its place and the pin connected trusses found theirs. And today we use both types, each in its appropriate place.

By 1900, reinforced concrete building appeared; we were all developing the theory and practice of reinforced columns and beams. Because of inadequate theory and of improper construction methods, many concrete structures collapsed. But the advantages of this new type soon made themselves felt, and reinforced concrete bridges and buildings took their places alongside steel buildings, pin connected bridges, and riveted bridges.

Great were the lamentations when the spirally reinforced columns and the flat slab made their bow into the structural world, but nothing of a very serious nature happened. Perhaps a few structures went wrong, but spiral columns and flat slabs are here to stay.

So it went, pins, rivets, reinforced concrete, all working together peacefully. Yet well do we remember the difficulty with which building codes were modified to permit concrete buildings. In many cities, reinforced concrete structures were built several years before their building codes were modified to admit the new type.

And now comes something new, but already old enough to have received a thorough trial. The welded steel joint puts in an appearance, starting about 1915, so far as welded structures are concerned. Welded rails have been in common use since about 1887. The welded joint suitable for many types of steel structures is here to stay. Let's recognize this fact, see that proper methods of design are applied, and that good welders are employed by responsible persons to execute our designs.

Influence of Joints on Truss Members

Each new joint has resulted in new shapes for main truss members. Pins required eye bars, looped rods, adjustable counters, heavily reinforced ends for compression members. Rivets eliminated the eye bar and looped rods, and demanded angles, channels, built-up members, gusset plates. Then came Bethlehem and Carnegie shapes to minimize the use of rivets, to reduce shop work, and to simplify steel construction generally. Now designers are asking for new and more convenient shapes for welded trusses. Trusses with all members made of cylindrical pipes have been tried. And how much did those members remind me of the old Phœnix columns with their circular segmental sections, their numerous rivets, their difficult joints! Nowadays welded connections to old Phoenix

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columns could be made with ease. We do not have to "buck up" the rivet in a welded joint.

Some new shapes are now needed to completely utilize the several benefits inherent in a welded joint. And as some good minds are at work on this problem, doubtless new structural shapes will be forthcoming.

Electric Welding

Electric Welding may be classified as (a) resistance, (b) arc, (c) arc and hydrogen.

(a) In resistance welding, the current flows from one electrode through two pieces to be welded to the other electrode, pressure being applied through the electrodes, to hold the pieces together during the heating process. Spot welding is a common type of resistance welding.

(b) Arc welding utilizes the electric arc to produce heat sufficient to melt two pieces together, the arc being formed between one electrode, (generally a wire, held in an electrode holder in operator's hand, or in an automatic welding machine), and the parent metal, which is the other electrode. The junction between the two pieces being welded is fused, both pieces as well as the welding wire being melted, thereby joining the two pieces together.

(c) Arc and hydrogen welding is the newest and includes two methods. First, dissociation of the hydrogen molecule into atoms by bathing the arc between two tungsten electrodes in hydrogen. As these separated atoms reunite, great heat is produced to melt together the two pieces being welded. Second, enveloping the ordinary arc in a hydrogen bath, the arc here being formed between the melting wire electrode and the pieces to be welded.

Functionally, electric welding is classified into, first, spot welding, the usual form being the resistance type; second, tack welding or intermittent, whereby two pieces are arc welded by depositing fillets at intervals along their joint of contact; third, continuous fillet welding, where the fillet is deposited continuously along the joint.

Extent of the Use of Welding

A list of welded structures I have prepared includes 35 buildings, of which the heaviest and largest is Building Number One at West Philadelphia Works of the General Electric Company now (March, 1928) being constructed; 25 miscellaneous structures such as towers, gas-holders, oil tanks, four truss bridges and a half through plate girder railroad bridge; 15 ships, barges, pontoons, etc. In many large industrial works, the use of castings has to a great extent been replaced by welded structural steel. This substitution results in stronger and lighter members, produced from cheaper material, at great saving in cost.

Some Problems to Be Solved

After having been identified with getting the building permit, with preparation of designs and shop drawings, with shop fabrication and erection of the West Philadelphia building, I select the following problems as worthy of mention:

THE WEST PHILADELPHIA BUILDING DURING CONSTRUCTION

1. Revision of state and municipal building codes to include positive permission to authorize welding. Many cities permit welded structures, others do not. Detroit and Philadelphia are among the larger cities where some permits have been granted. In the latter city, after a thorough investigation, the Bureau of Building Inspection granted a permit to the General Electric Company for the West Philadelphia Building Number One. The officials of the Bureau realized, as they did earlier when reinforced concrete came into use, that here is a new useful tool, which if properly used, produces safe and strong structures. Where the taxpayer can make an economic saving in his building by adopting welding it is the city's duty to permit him to effect that saving, provided safety is assured. Welding has left the experimental stage; it has arrived.

2. The second concern of engineers is to exercise supervision over employment of welders, to the end that they be given simple yet searching tests to show proficiency, before being allowed to weld important work. A capable inspector should test the applicant's ability to hold a steady arc of short length; to deposit a fillet free from gas; to recognize, in the behavior of the arc, a current too strong or too weak, or a voltage too high for thin steel or too low for thick work; to secure adequate penetration on each of the two pieces being welded; and to deposit fillets on vertical as well as horizontal surfaces.

Inasmuch as templets are largely eliminated from the fabricating shop, some substitute such as jigs must be employed to quickly assemble the various parts of trusses or girders. The reduction in shop costs can start on the working drawings by specifying consecutive progressive dimensions to real, not imaginary, working joints.

4. Automatic welding machines with two heads to deposit two fillets, side by side or in tandem as needed, effect reductions in welding costs. Such machines should be utilized where practicable. For example, in welding the longitudinal joint of a cylindrical pipe line, the two welding heads work in tandem; whereas in welding a flange plate to a plate girder web, the heads move side by side.

Welding the General Electric Company's Building

Both at the Trenton shop of the American Bridge Company and at the West Philadelphia site, single operator motor generator sets were used,—five at Trenton, two at West Philadelphia. This means that the 989 tons of steel for this building were welded by five men at Trenton, and that the field welding was accomplished by two men. Of the 989 total steel tonnage, 785 tons passed through the welding shop. Each welding machine consists of a generator, control panel, motor, starter and reactor, assembled on a welded steel frame. At West Philadelphia, each welding unit was made portable, by mounting on a simple hand truck.

When depositing a ⅜″ x ⅜″ triangular fillet, the arc voltage was about 23 to 25, and current 175 amperes. G. E. 3/16″ diameter electrodes, type F, were used. Direct current was used subject to control by the operator, the current being changed by shifting the brushes. Low voltage was used for light work and higher voltage for heavy work.

In considering the depositing of these fillets by shop men, the draftsman should remember that generally speaking the wire electrode is held so as to bisect the angle between the two surfaces being welded. Consequently, structural details should be so shown on drawings as to permit the electrode being held in this position. Sometimes it is necessary to bevel the ends of angles to accomplish this.

At American Bridge Company's Trenton fabricating shop, trusses, columns, etc., were welded while lying on a series of skids consisting of 18″ I beams from 21′ to 24′ long, supported directly on steel plates resting on the shop floor, and connected together by a small steel bar welded to each beam. The positive terminal was bolted to the web of one of these beams while the negative was connected to the 3/16″ x 24″ electrode in the holder held in the operator's hand. See Fig. 1.

The 206 skylight trusses were nearly all alike and were assembled by clamping the truss members in jigs, which expedited the process greatly. Jigs for such trusses can be made by welding flat plates to the skids, then welding small clip angles to the plates, so that, when the truss members are laid against the clips, or blocks which bear against the clips, the truss members are quickly assembled, clamped and welded. Working reference points are prick-punched on the plates.

The main roof trusses were assembled by first laying and clamping the bottom chord against a long 8″ x 8″ straight angle welded on the top of the I beam skids, a series of wooden blocks being interposed to produce the camber. The top chord, verticals and diagonals were then clamped in position. All welding that could be done was then laid, the truss turned over and the remaining welds deposited.

Overhead welding was avoided wherever possible. While many welders can weld overhead with excellent results, others cannot.

All welds specified on shop drawings, were marked on the assembled steel, and it is interesting to compare the lengths of fillets specified on drawings with actual lengths obtained on the trusses. The results for one of the 78′ aisle trusses, T14, are shown in accompanying table No. 1. The following may be observed:

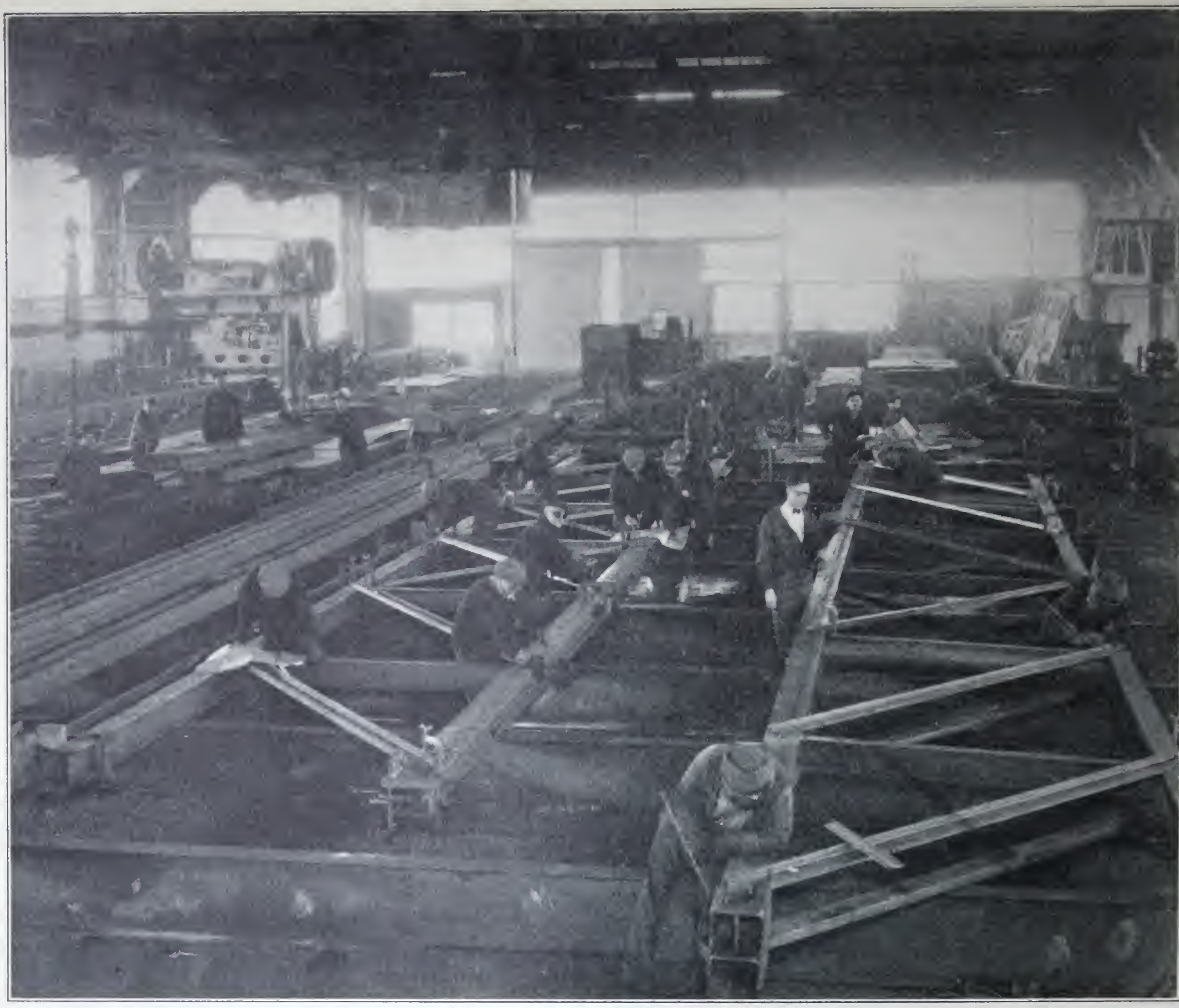

Figure 1. Shop Work at Trenton

Truss	Upper End		Lower End		Total Both Ends	
Member	Drawing	Truss	Drawing	Truss	Drawing	Truss
U_0L_1	30	33¼	30	31½	60	64¾
U_1L_2	30	32¼	30	34	60	66¼
U_2L_3	18	20¾	18	21½	36	42¼
U_3L_4	6	8½	6	7¼	12	15¾
$U^1_3L_4$	6	9¼	6	8¼	12	17½
$U^1_2L^1_3$	18	21	18	20	36	41
$U^1_1L^1_2$	30	33	30	32	60	65
$U^1_0L^1_1$	30	33	30	33	60	66
U_1L_1	20	19¼	20	18½	40	37¾
U_2L_2	20	19½	20	18¼	40	37¾
U_3L_3	13	11½	13	13	26	24½
U_4L_4	13	12	13	12¾	26	24¾
$U^1_3L^1_3$	13	12	13	12½	26	24½
$U^1_2L^1_2$	20	18	20	18½	40	36½
$U^1_1L^1_1$	20	18½	20	18¾	40	37¼
U_4L_3	6	10	6	8¼	12	18¼
$U_4L^1_3$	6	6¾	6	6¾	12	13½
Total					598	633¼

TABLE I

Comparison between lengths in inches of fillet welds on web members specified on drawing and actual measured lengths of fillet welds on arc-welded truss T-14.

1. Total length of actual is 633¼″, as compared with 598″ specified.

2. Maximum excesses of actual over specified for any one member are 5½″, or 46% of specified weld; and 6¼″, or 52%, for another fillet.

3. Maximum deficits of actual as compared with specified for any one member are 1½″, or 6% of specified weld; and 3½″, or 8.8%.

4. Individual fillets may be as much as 10% shorter than specified.

All of which shows inspection to be necessary; but also that maximum deficits are not sufficient to appreciably reduce the factor of safety.

Shop fabrication at Trenton and field erection at West Philadelphia each required about two months.

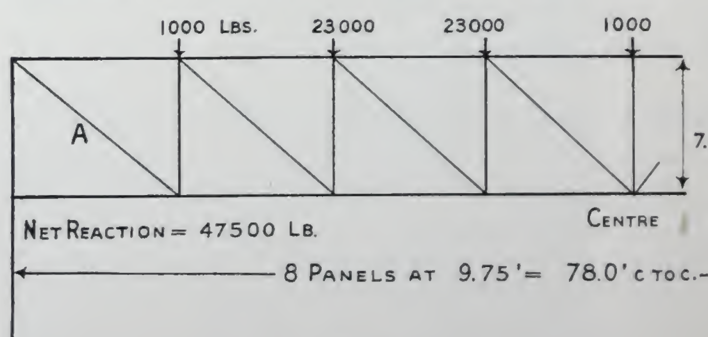

Figure 2

Some Details of Design

Example A. Given a roof truss, Fig. 2, having a top chord made of one 8″ H section with web hori-

zontal; proportion end diagonal A and fillets connecting it to top chord, using 16000 lb. sq. in. for tension in diagonal and 3000 lb. per linear inch as shearing value for ⅜″ x ⅜″ triangular fillets; truss symmetrical about center.

Tension in end diagonal $= \frac{47500 \times 12}{7} = 81400$ lbs.

Net area required in end diagonal $= \frac{81400}{16000} =$ 5.1 sq. in.

Use 2-5″ channel's 9.0 lbs. = 5.26 sq. in.; gross and net.

Length of ⅜″ fillet required $= \frac{81400}{3000} = 27.1''$.

Use 4 fillets each 7″ long, Fig. 3.

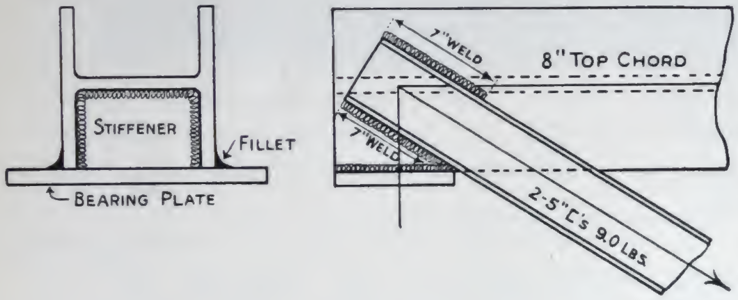

Figure 3

The simplest way to indicate the 7″ fillet of Fig. 3 on a shop or field drawing is by the note 7W or 7FW respectively.

Example B. Given a welded plate girder, span 40′ c. to c., uniform load of 4000 lbs. per linear foot over entire span, assuming web 40″ x 5/16″; proportion tension flange, using 16000 lb. sq. in.

Maximum moment $= \frac{4000 \times 40 \times 40 \times 12}{8} =$ 9,600,000 in. lb.

Required net area $= \frac{9600000}{40 \times 16000} - \frac{1}{6} \times \frac{5}{16} \times 40 =$ $15.0 - 2.1 = 12.9$ sq. in.

Use one plate $16'' \times \frac{13''}{16}$, or two plates to give equivalent area.

Notice that this simple calculation reveals a double saving; first, in that no allowance need be made for rivet holes; and, second, that as no vertical row of holes exists in the web, the web equivalent for the welded girder is larger and the flange correspondingly smaller than in the riveted girder. The greatest saving can be obtained in welded girders, however, by using, instead of angle stiffeners, flat bars with one edge welded directly against the web, thus eliminating fillers as well as the legs of angle stiffeners, which lie parallel with the web, Fig. 4. The West Philadelphia building has no plate girders but this example has been introduced here to illustrate an economy resulting from welding.

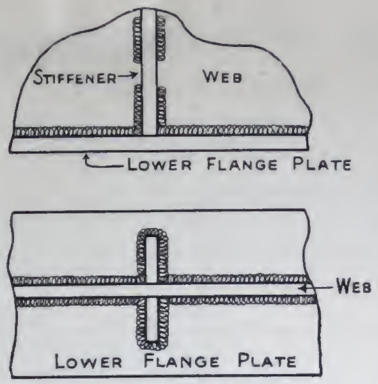

Figure 4

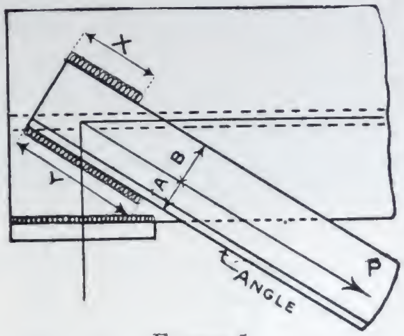

Figure 5

Example C. Given an angle in direct stress such as tension, Fig. 5, to find the proper lengths of fillets to connect the angle to connecting piece; using tension at S lb. sq. in. and longitudinal shear on fillet at f lb. per linear inch.

Using .6 as factor to allow for eccentric loading, the required area of one angle $= \frac{P}{.6S}$ where P = pull in angle, and S = allowable unit tension. By taking moments about outer edge of angle leg the length of fillet required along back of angle = Y = $\frac{PB}{(A+B)f}$ and length of fillet X for outer edge of leg = $\frac{PA}{(A+B)f}$. As they should, these add to $\frac{P}{f}$

General Description of the Building

Although Building Number One is first in number, it is not first in order of construction, as several other buildings have been in use for some time. This latest structure, differing from the others in that its steel frame is arc welded, consists of a head house, or transept, parallel with Elmwood Avenue, and two main aisles at right angles thereto. The headhouse is 78′ wide and 171′ long, with vertical clearance of 43′. One aisle is 59′ wide and 474′ long, with vertical clearance of 34′ from ground floor to lower chords of roof trusses, while the other aisle is 79′ wide and 474′ long, with a reinforced concrete gallery floor, in all but the two westerly bays, nearly midway between the ground floor level and the lower chords of trusses. To be exact, the gallery floor is 20′ above the ground floor, and is connected with a similar gallery in existing building Number Four by an enclosed passageway. These aisles and the headhouse comprise a building approximately 138′ wide by 552′ long, with total ground area of 80150 sq. ft.

Concrete spread footings in clayey soil serve as foundations for brick walls, steel columns, and for the reinforced concrete columns supporting the gallery. The ground floor throughout consists of a wooden block surface on a concrete base on cinder fill,

and the gallery floor is also wooden block on flat slab construction. Outside walls are of brick; and the roof, gypsum slab cast in place. The building will be well provided with natural light from unusually large windows in all external walls, and from four lines of sawtooth skylights, two lines in each main aisle.

Steel Framing

The steel frame consists of steel columns, made of new Carnegie beam types, between which are welded transverse trusses of the Pratt type with parallel chords in the two main aisles, but with inclined top chords in the headhouse trusses. The latter trusses have spans c. to c. of columns of about 77′, and vary in depth from 8′ 4″ at one end to 6′ 4″ at the other to carry the sloping roof of the headhouse. One main aisle has roof trusses 7′ deep with 6 panels each 9′ 9″ making the span 58′ 6″. [See Figure 8, page 13.] The other main aisle has 78′ trusses 7′ deep with panels at 9′ 9″. As all truss chords in the main aisles are horizontal, and as the main purlins rest on the top chords of these trusses, roof slopes for drainage are provided for by sloping the secondary purlins.

The following data show the very considerable amount of saving in weight of steel resulting from the use of welded in place of riveted trusses. For 78′ span, estimated weights of welded and riveted trusses were 9200 lbs. and 13200 lbs. respectively, a saving of 4000 lbs. or 43 percent of the welded truss; for 58′ 6″ span, estimated weights of welded and riveted trusses were 5000 lbs. and 6800 lbs., respectively, a saving of 1800 lbs. or 36 percent of the welded truss.

In general each column had a 1½″ cap plate welded directly to its web and flanges, but the 1½″ base plate was in most cases not welded to the column. Each anchor bolt was connected to each column by passing between the column flange and a short angle standing vertically with outer edges of both legs welded to the face of column flange. The anchor bolt nut bears against a plate washer resting on upper end of the angle. [See Figure 4, page 10.]

The bays in main aisle were 24′ to 25′ while those in the headhouse were from 25′ 7″ to 29′ 3″.

The design of roof trusses is characterized by the use of one 8″ Carnegie beam for each top and bottom chord with flanges vertical; to the outer surfaces of which are welded channel diagonals and to inner surfaces 7″ I beam vertical. [See Figure 7, page 12.] By thus welding diagonals and verticals directly to chord flanges, the use of over 1200 gusset plates was avoided in the trusses. Another feature is the absence of lattice bars, not only in all trusses, but generally throughout the building. In only two members in the entire building were lattice bars used. Figure 6 shows photograph of finished joint.

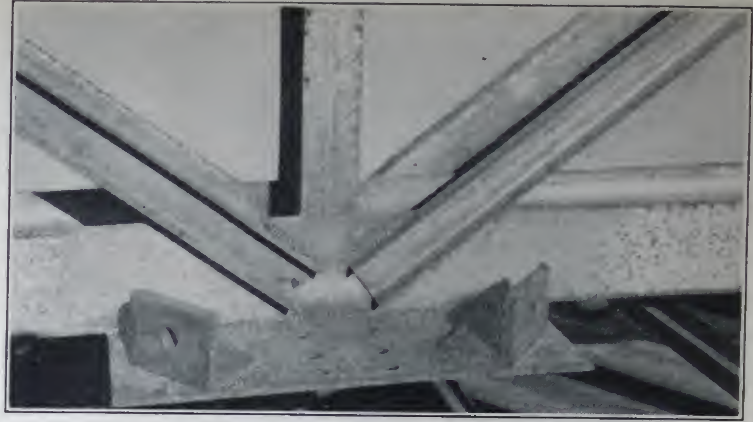

FIGURE 6—TRUSS DETAIL

The fillet welds, of triangular cross section with base and altitude generally ⅜″ each, are subjected to a longitudinal shear, and the unit shearing stress used in the design is 3000 lbs. per linear inch for ⅜″ fillets. As the critical shearing section for a ⅜″ x ⅜″ triangular fillet is .265″, this corresponds to a shear 11300 lbs. per sq. in. In accordance with many tests made, this represents a factor of safety of at least four.

A roof load of 60 lbs. per sq. ft. was used in the design, divided equally between live and dead, applied not only to the level portions of the roof but also to the sloping surfaces of skylight monitors. Unit stresses in steel members were those of the Philadelphia building code.

Horizontal bracing, consisting of diagonal rods with Carnegie beam struts, is provided in plane of truss lower chords in all bays of the headhouse and in the 58′ 6″ main aisle where cranes operate, but in the 78′ main aisle with concrete gallery, lower chord bracing is used in only four bays. Horizontal top chord bracing is provided in four bays of both main aisles and in two headhouse bays. Vertical longitudinal bracings also were used in several bays to resist crane stresses.

Cranes

Provisions were made for the following cranes. In the headhouse, 10-ton bridge cranes with span of 73′ 5½″; in the 59′ main aisle, 10-ton bridge cranes with span of 55′, and 2-ton wall cranes on either side; in one end of 79′ aisle, one 5-ton bridge crane with span of 46′ 2″. All bridge crane runway girders are Carnegie beams supported generally directly on top of auxiliary crane columns. The only exception is at intersection of headhouse and 59′ main aisle where the 10-ton headhouse crane girders are supported on a deep welded truss, carrying not only the crane but roof and clerestory loads as well.

The wall cranes are supported on brackets carried by the auxiliary crane columns and main columns. And as these cranes exert horizontal forces

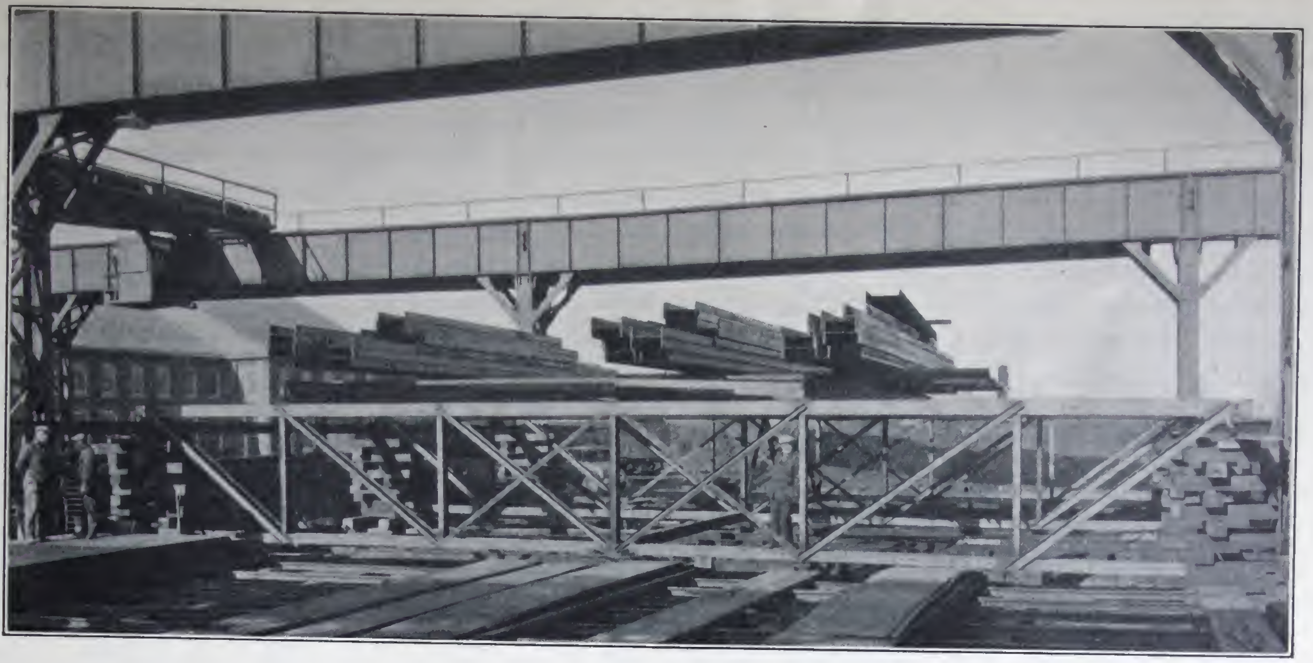

Figure 7—Set-up for Truss Test

at both upper and lower levels, as well as vertical forces at the lower level, their runway supports are not simple.

In the headhouse are three interesting structural features; first, supporting one transverse headhouse 77-ft. truss on two longitudinal wall trusses, at connection with this new building to an existing adjacent one; second, supporting one end of one transverse headhouse 77-ft. truss, at the center of a 58′ 6″ truss, at intersection of headhouse and one main aisle; third, supporting headhouse crane runway girders by welding them to cantilever ends of two needle beams supported by 58′ 6″ trusses, at intersection of headhouse and main aisle.

Tests

The design of this building was based in part upon tests made by Rensselaer Polytechnic Institute for the General Electric Company. The plates were of such sizes and thicknesses that, at the ultimate loads, the stresses in the plates were much below the elastic limit. For specimens in tension the 3/8″ x 3/8″ triangular fillets of varying lengths gave an average longitudinal shearing strength of 13300 lbs. per linear inch of fillet. Whereas, compression specimens with varying length of 3/8″ x 3/8″ fillets gave from 17800 to 15800 lbs. ultimate shearing strength per linear inch of fillet. See Table II for typical test data. It is apparent that the 3000 lbs. per linear inch used in design for this building gives ample security.

At Trenton, three 58′ 6″ trusses were temporarily erected for testing, being supported and braced and erected in the same relative positions as they were to occupy finally in the completed structure. Test loads consisting of I beams were then applied to the top chord joints through purlins until the center truss carried a total load equivalent to 1½ times the load

Designation	Fillets Number	Fillets Length In.	Ultimate Strength Total Lb.	Ultimate Strength Lb./Lin. Inch
8A-1	4	3	173,300	14,420
2	4	3	164,100	13,700
3	4	3	154,400	12,880
Average				13,600
9A-1	4	4	212,080	13,290
2	4	4	215,100	13,450
3	4	4	191,600	11,990
Average				12,900
10A-1	4	5	276,580	13,830
2	4	5	268,300	13,410
3	4	5	262,100	13,110
Average				13,420
11A-1	4	6	317,000	13,210
2	4	6	316,900	13,210
3	4	6	330,300	13,750
Average				13,390

TABLE II

Some test data for determining strength of welds.

for which it was designed. As the total design dead load was 30 lbs. per horizontal sq. ft. and the live the same amount this test load was equivalent to the full dead plus twice the live load. See Figure 7. [Also, Figure 3, page 16.]

The vertical deflection at center top chord joint checked very closely with the calculated deflection. Deflections were observed (Table III), till the truss

Total Loads on Truss Panel 1	Panel 2	Panel 3	Central Deflections Loads Increasing	Central Deflections Loads Decreasing
4050 lbs.	4050 lbs.	3430 lbs.	0.00″	0.00″
7850 lbs.	7850 lbs.	7230 lbs.	0.11″	0.10″
11650 lbs.	11650 lbs.	11030 lbs.	0.20″	0.20″
15450 lbs.	15450 lbs.	14830 lbs.	0.31″	0.30″
19610 lbs.	19610 lbs.	18990 lbs.	0.41″	0.41″
23770 lbs.	23770 lbs.	23150 lbs.	0.52″	0.52″
28260 lbs.	28260 lbs.	27500 lbs.	0.67″	0.66″
32750 lbs.	32750 lbs.	32970 lbs.	0.78″	0.78″
35000 lbs.	35000 lbs.	35220 lbs.	0.89″	0.89″

TABLE III

Test of Welded Truss for Deflection.

was carrying the design load, when the central deflection was .52 of an inch; then continued till the truss supported the design dead load plus double the design live load, at which time the deflection reached .78 of an inch. As the loads were removed the deflections decrements were almost exactly equal to the increments under increasing loads. Till finally upon removal of all applied loads, the truss occupied exactly its initial position, showing that no slipping in joints took place under the test. Calculations for central deflections are shown in Table IV.

Calculated Central Deflection for Truss $= \Sigma \frac{STL}{AE}$

Bar No.	S Kips	T Lbs.	L Feet	A Sq. In.	$\frac{STL}{A}$
1	67.5	.70	9.75	9.1	50.6
2	94.7	1.40	9.75	9.1	142.0
3	121.9	2.10	9.75	9.1	274.0
4	121.9	2.10	9.75	9.1	274.0
5	108.8	1.40	9.75	9.1	163.0
6	54.4	.70	9.75	9.1	40.7
7	67.5	.70	9.75	7.06	65.2
8	94.7	1.40	9.75	7.06	183.0
9	108.8	1.40	9.75	7.06	210.0
10	54.4	.70	9.75	7.06	52.5
11	83.0	.86	12.0	5.26	162.9
12	48.5	.50	7.0	4.80	35.4
13	33.5	.86	12.0	2.38	145.2
14	19.5	.50	7.0	4.43	15.4
15	33.5	.86	12.0	2.38	145.2
16	29.0	1.00	7.0	4.43	45.8
17	16.2	.86	12.0	2.38	70.3
18	39.0	.50	7.0	4.43	30.8
19	67.0	.86	12.0	4.24	163.2
20	39.0	.50	7.0	4.43	30.8
21	67.0	.86	12.0	5.26	131.1

$$\Sigma \frac{STL}{A} = 2431.1$$

TABLE IV

$$\text{Central Deflection} = \Sigma \frac{STL}{AE} = \frac{1000 \times 12 \times 2431.1}{30{,}000{,}000} = 0.97''$$

S = Stress due to loads on truss.
L = Length of truss member.
T = Stress due to load of unity at center.
A = Gross cross sectional area of truss member.
E = Modulus of elasticity.

Before any welder was allowed to do work on this building, either at the fabricating shop or at the building site, he was given certain tests by a capable welding inspector, and his work was subsequently watched by the inspector. A comparison in accomplishment was made at Trenton by having each of the five welders make two test pieces consisting of two 6″ x ⅞″ main plates spliced by two 5″ x ½″ plates with 10″ of ⅜″ x ⅜″ fillets on each side of the splice. These 10 specimens were then tested (Table V), and

Designation	Welders No.	Average Total Length of Fillet	Ultimate Load	Strength Lbs. Lin.In.
3B	3	10″	120,700	12,070
3A		10″	132,670	13,267
1A	1	10″	127,850	12,785
1B		10″	120,630	12,063
2A	2	10″	120,570	12,057
2B		10″	113,930	11,393
4A	4	10″	115,510	11,551
4B		10″	111,580	11,158
5A	5	10″	105,660	10,566
5B		10″	101,370	10,137

TABLE V
Report on Tests of Welded Test Specimens

showed that, while the two specimens made by any individual welder gave remarkably close results, the variation between the lowest average and the highest, i. e., between poorest and best welder, was about 22% of the lowest. The lowest values, however, were practically the same as those used in design; it is the lowest value that controls, not the average. Of course, two specimens per welder are not many, but taken in connection with other tests and accomplishments of these five shop welders, it is believed the specimens represent shop variations. These are no greater than exist in concrete tests or in riveted joint tests.

Welding an Industrial Building*

By ANDREW VOGEL,
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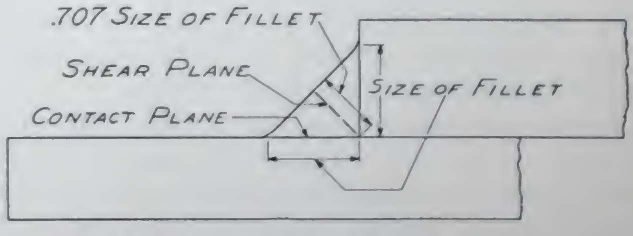
THE design of trusses and other structural features of the building at West Philadelphia required the determination of unit stresses to be used for various types of welds. Data for these units were sought from the literature on the subject, but it was found almost invariably that the specimen itself either distorted badly or broke, before the welded joint had reached its maximum capacity. Obviously that type of test specimen was wrong. It thus became necessary to design a test specimen that would develop the welded joint to its ultimate capacity, without the specimen itself being stressed beyond its



DETAIL OF FILLET WELD

FIGURE 1—CROSS SECTION OF WELD USED IN TESTS

*Second paper presented before the Philadelphia Section of the A. S. C. E. at the Engineers Club of Philadelphia, March 14, 1928.

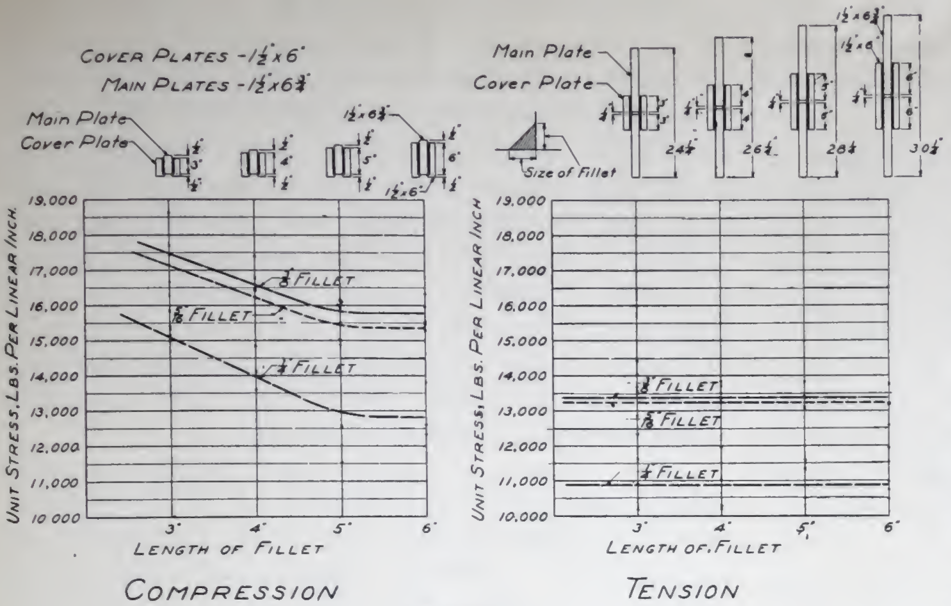


Figure 2

elastic limit. In other words, the welded joint must be broken before the specimen became distorted. A group of twenty-four specimens were so designed for welds of 3″, 4″, 5″ and 6″ lengths, with the welds designed to fail in shear on a plane perpendicular to the long side of the triangular cross section of the weld. Figure 1, showing a detail of a fillet weld, indicates that the shear plane, being the weakest plane across the weld, is in pure shear, if no secondary stress occurs at the joint or in the test specimen, due to deflection or distortion of the specimen. Of the 96 welds in this series of tests, 93 broke along the shear plane, and three broke along the contact plane. In each of the three cases where the specimens failed at the contact plane, bits of the parent metal were torn out, showing that thorough fusion had taken place; and in these three cases, the unit stresses developed were directly comparable or equal to those where failure was in shear.

The diagrams on Fig. 2 indicate the uniformity of these tests. The tests were divided into two parts, the compression group and the tension group. The compression group gave higher values than the tension group; and this has been explained by the fact that the compression group were short chunky specimens, in which bending could not occur due to stresses in the specimen, while the tension group were of necessity long specimens, in which slight bending would occur in the outer plates even though stresses were within the elastic limit.

From the tests so made, it was decided that welding with a minimum contact distance of 5/16″, and a maximum contact distance of ⅜″, the variation being specified to allow for the inaccuracies of workmanship, permitted a unit stress of 3,000 pounds per linear inch. The tension tests indicated a range of 11,570 to 14,040 pounds per linear inch, with an average of 13,055 pounds per linear inch, for welding of 5/16″ contact distance; with ten of the twelve specimens having a value of 12,300 pounds or more per linear inch, and the other two, 11,570 and 11,930 pounds per linear inch.

It will be observed that, basing the unit stress on the values obtained from the tension specimens alone, a factor of safety of four or better is obtained in practically all cases. If, on the other hand, the compression specimens are considered, a factor of safety of five or better is apparent in all cases. It is, however, necessary to be conservative in any new type of design, and it was for this purpose of being conservative, together with the thought that a new method must not be handicapped by too low unit stresses, that the value of 3,000 pounds per linear inch was adopted.

Here, then, are the two essentials of truss design; first, the fact that the stresses can be readily calculated for the various members and joints; and second, the unit stress to be used for the design of the joints.

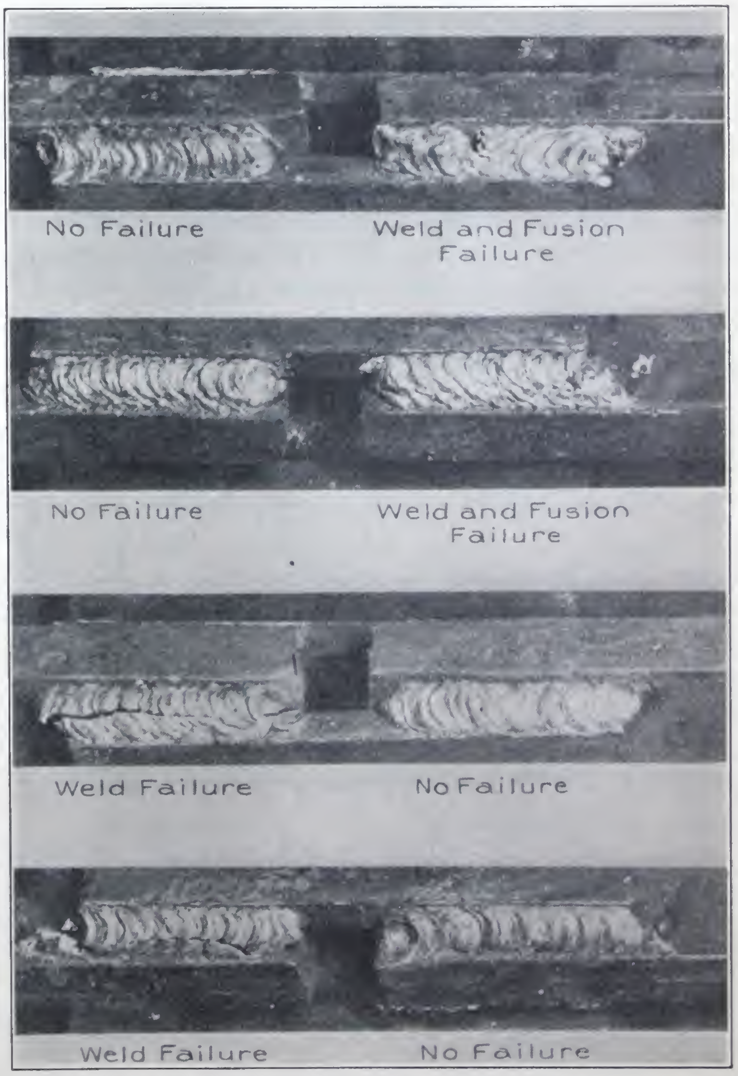


Figure 3

The unit stresses used for the design of the members of the trusses were, of course, those regularly used in the design of structural steel, in accordance with standard American practice.

Since the former tests were made, a number of tests have been conducted of the work performed by average workmen, and Fig. 3 illustrates all four edges of a test specimen as it was rotated in front of the camera. This specimen illustrates typical shear plane and contact plane failures, but the shear failures are marked "weld failure," and the contact plane failures are marked "fusion failure." It is interesting to observe that, though these welds appear rough and not up to the high standard that can be attained by expert welders, still the test developed an ultimate load of 13,267 pounds per linear inch of weld.

Experience with welders has shown that, while natural aptitude is an important factor, these men must be trained to their duties as in any other trade, and that there is as much difference between an apprentice or student welder and the experienced welder as there is between the apprentice and the experienced workman in any other trade. Individuality will be largely eliminated, since experience shows that trained workmen and proper methods of welding produce uniform results. Everyone is familiar with the change in quality, and the trend toward higher quality and uniformity in concrete work as specifications were changed from the typical 1:2:4 concrete to specifications which included the water content in the general form of 1:1:2:4. Just so, it is hoped that, eventually, the technique of welding, as much as that in concrete work or in any other of the building trades, will become standardized, and that it will be possible to make every weld a good weld.

The introduction of machine welding in the shop fabrication of steel construction is as certain as it has been in other industries throughout the country. There are available continuous electric arc welders; intermittent arc welders, which weld a specified distance, skip a specified distance, and then weld again and repeat; and also double header welders, which weld two lines at once. Then there is the portable welding machine, which in the future will probably be controlled either by guides or some other device, enabling workmen rapidly and accurately to weld practically all manner of joints in structural steel work.

There are two main incentives to welding, saving in cost, and silence; and economy applies with particular force to trusses. In the design of trusses, it is possible to save in all tension members a strip of metal the full length of the member and of the same width as the rivets. It is also possible to eliminate gusset plates, and these two items alone, in the case of the West Philadelphia building, produced a saving of 12 to 15 per cent in steel tonnage. The connection of members is simpler, inspection and painting are simpler and easier, and economical rolled sections can be used. Of course, as welding design changes, it will be necessary to use other types of sections than those now used for riveted work, but assurance has been given that these new sections will be made available as needed. Simplified design and drafting are also elements of the reduced cost; but it will be necessary to change, at some expense, shop equipment, and shop and erection methods, though these also eventually will be a source of additional saving.

The silence of welding steel structures is extremely noticeable. The building at West Philadelphia (containing nearly 1000 tons of steel) was erected in a residential neighborhood without the neighbors being aware that anything out of the ordinary was occurring. Had it been a riveted building, the ceaseless racket of the riveting hammer would have apprised the residents of the neighborhood that the building was under construction and would have

Figure 4—Details of Column Anchorage Showing Equal Leg Angles Welded to Steel Column

kept them fully informed that it was progressing until the completion of the steel frame. Those acquainted with the Hotel Benjamin Franklin in Philadelphia, remember the annoyance suffered there when a building near by was riveted, and those acquainted with downtown New York often experience the difficulty of hearing on the telephone when riveting is going on near by. Recently an addition to a hotel building was contracted for on the basis that the field work must be welded, with the object not only of saving its guests the annoyance of the noise of riveting, but also of preventing loss in revenue to the hotel due to the departure of guests to escape the racket.

There are certain interesting details in connection with the West Philadelphia building mentioned that will bear examining somewhat closely. Sometimes a particularly interesting detail illustrates economy of welding and also correct methods of designing. Fig. 4 shows the method of anchoring columns to bases. The bending moment at the foot of the column was calculated, and the size of anchor bolts and base plates determined, to care for the bending moment and direct loads, in the same manner as required for reinforced concrete construction. With the stress in the anchor bolt determined, the next calculation consisted of dividing this stress by 3,000 lbs. per linear inch to determine the number of inches of weld required to attach properly the equal leg angle to the column. The cover plate consists of a bar with a hole punched through it. This is placed on top of the angle and then the nut is drawn down tight. The result is one of the most economical forms of column connections. If horizontal forces are applied at the foot of the column, it is necessary to weld the column to the base plate and the base plate must then be properly anchored or secured to the concrete foundation, by the ability of the anchor bolts to withstand shear or by a sufficient amount of concrete properly bonded to the foundation.

The development of the trusses was an interesting study. Fig. 5 illustrates a riveted truss in Building No. 4 at this same plant and a welded truss of equal capacity. It will be noted that on the welded truss some unusual chord sections are indicated, that is, the use of one-half of an 18″ x 87½ lb. Bethlehem girder for the top chord and one-half of an 18″ x 49 lb. Bethlehem I-beam for the lower chord. It was proposed to split these members by the use of the torch, thus bringing in one more of the newer tools of building construction. This truss also illustrates the use of tees for web members welded to the webs of the large size tees obtained from girder and beam sections.

A study of this design naturally led to its later use, as shown in Fig. 6, for a series of trusses of 64′ span to be used at another plant. Again chord sections are obtained by splitting large rolled sections. In this case the upper chord consists of one-half of a 16″ × 58 lb. Carnegie beam, while the lower chord consists of one-half of a 16″ × 35 lb. Carnegie beam. The web members are, however, single angles. The detailed calculations of one of the joints is given, in order to illustrate the method of applying the proper amount of welding on each side of a single angle so that the center of gravity of the angle corresponds with the center of gravity of the amount of welding used, that is, moments about the center of gravity of the angle, produced by the amount of welding used, were made equal, so that no unbalanced moment would be transmitted to the member, or unequal unit stresses applied to the welds. The example gives the method in such detail that it is unnecessary to describe it.

Fig. 7 illustrates the development of the Pratt truss used in the building. A riveted design of the usual type was first prepared, and then a welded design was prepared, following the same truss lines. The riveted truss required gusset plates and extra connection angles, resulting in the use of a large number of rivets. It is, of course, impracticable to connect directly together members in a riveted truss, thus necessitating the use of gusset plates. In the welded truss, however, members are directly welded to each other, so that one unit of connecting material per-

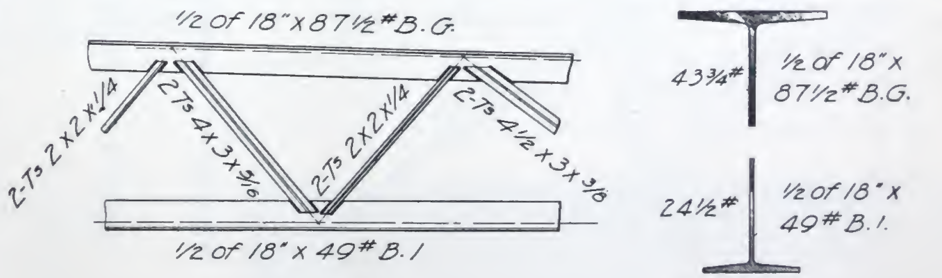


Figure 5—Comparative Welded and Riveted Designs

made of rubber, with lines drawn on them in India ink, and the specimens "welded" to a base plate of rubber with a high quality glue along the edges, indicated by the semicircular hachure lines. Five typical connections are shown, and the first three, starting from the left, show welds symmetrical about the center line of the member. There is little of interest in these specimens, because the center lines of the specimens remain straight lines. The two at the right, however, indicate unsymmetrical welds. In both cases, the specimens are distorted. Curved center lines indicate that additional stresses were set up in the specimens, due to the asymmetry of the connections. These studies, of course, are crude and not quantitative, but are shown to indicate that various methods of investigation are being used.

It is fitting at this point to state that the results at West Philadelphia have amply justified the expectations. The work has been accomplished with much less difficulty than anticipated, but, of course, the usual problems of developing a new method had to be met and conquered. More buildings are to be welded, and on these buildings, the accumulated experience from the first building will be used to great advantage. The General Electric Company will continue to weld buildings, and it will undoubtedly find more and more economies. It has already been found that there is a saving of material in the weight of steel required for trusses, a simplification in the fabrication of columns, a reduction in floor space required for shop fabrication, and a simplification in design and drafting room practice. In addition, there is the great boon of silence, which is becoming measurable in dollars, as evidenced by the addition to the hotel previously mentioned.

DISCUSSION

MANTON E. HIBBS: I feel sometimes I am like a family doctor to the building trade of Philadelphia. No matter what I say of the patient, the very next morning some young man will call at my office for further treatment.

I believe welding has a future, a very good future, but I don't believe it is so easy of calculation as the two previous speakers have so eloquently stated. However, one of the good points of welding is that it has drawn to the attention of structural engineers the subject of jointed connections, and has made further study possible in the same way as reinforced concrete has stimulated the steel industry to know better steel construction. I am quite sure we know welding today, by tests and experiments, far better than we knew reinforcing in 1902, when we constructed our first concrete building. At that time, our Bureau knew very little in comparison with what we know today, and the builder then knew even less.

Personally, I have been more concerned with how to judge good welding than with calculations. But after spending a morning in the shop of the General Electric Company at Schenectady, and carefully examining the welding done there, and in the afternoon seeing the welding done by the apprentices, good and bad, I made up my mind a good inspector could tell by his eye, good and bad welding.

I do not think welding will wipe out riveting, nor do I think it is a twin brother; but I do feel, there is an abundant and rich field for it. As far as our Bureau goes, I can frankly say, I would as lief accept welded trusses as riveted ones. For the present building discussed, the Bureau passed the work with the following regulations:

1. In trusses, gusset plates may be eliminated, provided suitable shapes be used so there is ample room for welding. In all cases, the Bureau reserves the right to reject any shape they feel is not suitable for welding or for the stresses to which the piece is subjected.

2. Allowable safe working stress for a ⅜"-5/16" fillet shall not exceed 3,000 lbs. per linear inch or fillet.

3. The deposit of any fillet once begun shall be continuous for the length of the weld.

4. The Bureau reserves the right to require tests on any welded structure, if the question of safety and stability is involved.

5. In welding, only competent workers trained in the art of welding shall be employed.

6. Before any welded structure is started, a competent trained engineer skilled in welding, must be furnished by the owner, and all work must be done under his personal supervision.

7. In equipment, only arc welding is to be used.

8. The *owner must take out the permit.*

J. H. EDWARDS: Some points of interest to the structural designer and fabricator may be added to those mentioned in the papers.

Until quite recently, those interested in the welding art as applied to structural steel were the manufacturers of welding apparatus. They were very optimistic, and saw great possibilities; but naturally, not being familiar with the various highly systematized structural shop operations, were somewhat misled by their enthusiasm when trying to adapt the welding process to the joining together of structural parts. It seemed to some of us that the problem could better be solved by the coöperation of apparatus manufacturers and users; the welding apparatus makers supplying the proper machines and instruction to operators, and the designers and fabricators of structural steel endeavoring to adapt their materials, and adjust shop methods to the new process.

Many of the structures or parts of structures so far welded were of the simpler type, and met only a few of the problems of structural design. The West Philadelphia

structure gave an opportunity to study about all the conditions usually met in industrial building work, such as, connection of new to old work, large-span roof trusses and lattice girders, provisions for traveling bridge cranes of fair capacity, and the unusual and difficult traveling jib crane supports.

The type of structure being comprehensive and attractive, the American Bridge Company undertook the construction of the structural frame, with a desire to try out the above mentioned coöperation. The General Electric Company engineers had made laboratory tests on joints, to determine unit strength of welds, and had also made a few studies of the steel frame arranged for welding, to take the place of the riveted design prepared by the architects. The unit stress for a standard single-run fillet weld, about ⅜-inch on its right-angle sides was fixed at 3,000 lbs. per linear inch. With this unit as a basis for determining the joints, and the architects' plans and specifications as guide to all general features to be met, the Bridge Company undertook the design with a view of obtaining these results:

1. Most advantageous use of the structural shapes available.

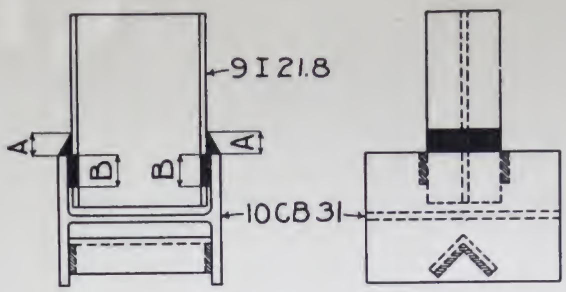
2. Elimination, as far as possible, of all the shop operations that might not be required in welding joints.

3. Assignment to the shop of the maximum amount of welding, thereby reducing the field welding.

4. Arrangement of details of field joints so that loads are carried in direct bearing, thereby reducing number of field holes, and also reducing field welding.

5. Avoidance of all rivets and use of bolts, except when and where they were necessary for erection, or where they might become permanent in minor connections.

The design of the column and girders presented no unusual features. The trusses offered the best opportunity to take advantage of welding, and were most interesting. After analyzing the studies made by the General Electric Company, and many others subsequently, made it was found that, by slightly rearranging the roof skylight, changing the depth and number of truss panels, (thereby reducing the number of members), what seemed to be the most economical design of truss was obtained. Practically all the saving in weight occurred in the trusses, where full advantage was taken of the gross section of members, gusset plates were eliminated, and members rearranged. While laboratory tests had given sufficient data for determining the strength of nearly all the types of joints used, it was desirable to supplement those already made by establishing values for some unusual ones. It is not always practicable or possible to arrange joints so that units determined by laboratory tests can be applied directly. There are inequalities and eccentricities that must be considered, and their behavior can best be judged by full sized tests of like or similar conditions of detail and load.



Test Results

Spec. No.	A	B	Max. Load Lbs.	Lbs. per Lin. In. Weld	Remarks
1	3/4	0	138930	16030	Welds torn from flgs. of 9" I.
2	5/8	1 3/4	203850		Partial Failure of welds and flanges of 9" I.
3	9/16	2 1/2	228850		Great distortion of 9" I and partial failure of welds.
4	0	2 11/16	133650	12520	Welds failed.

Results are the average of three tests per specimen.

Figure 1

The adaptability of the truss chords, of the Carnegie Beam Sections, of constant depth and parallel flanges, was evident, but the question of the strength of the joints of vertical web members to chords was not well settled. Tests were made of similar joints welded in various ways. The results are shown by the figures. Figure 1 shows the detail arrangement of tests, and gives a table showing the amount of welding and the strength shown by tests. Figure 2 shows the tests of type 3, where

Figure 2

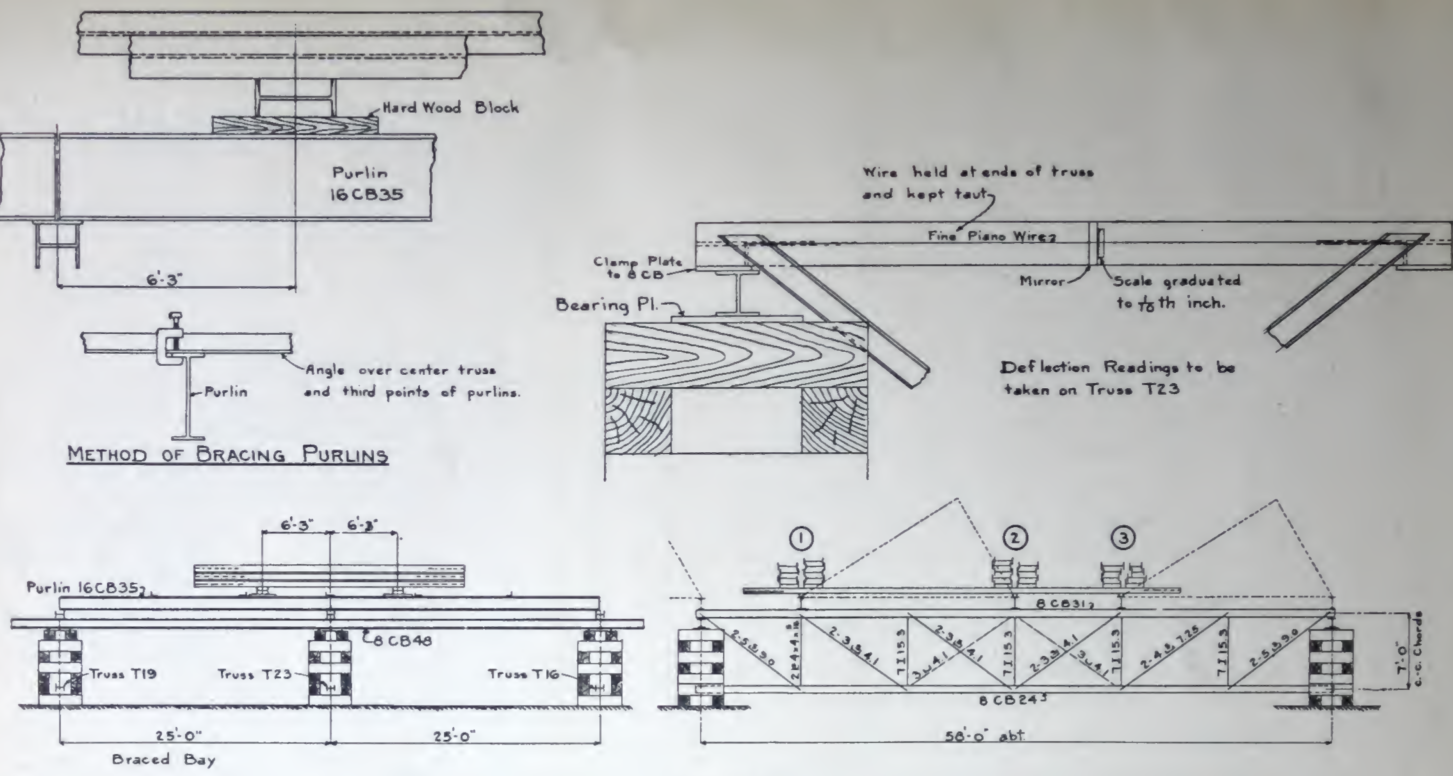


FIGURE 3

the welds were sufficient to cripple the I-beam member that represents the vertical web member of the truss. Other tests were made of special joints to make sure all conditions of stress were well considered. See Fig. 3.

The open-minded attitude of the Philadelphia Bureau of Building Inspection toward the permit for the welded structure was very gratifying. There was no prejudice or doubt shown. After having made a study of the application of the process, they were willing to permit the use of welding, with the restriction that work should be done under proper procedure by capable and responsible parties.

The economic waste encouraged by adhering to Building Code rules that have not kept pace with the development of the construction art is enormous. The reason often given for not permitting the use of any new materials or methods, that irresponsible parties may perform the work, and inspection may be indifferent, should not prevail. It puts a premium on ignorance and incompetence, penalizes the investor, and impedes progress. The reaction one notices when welding is suggested in connection with structural steel fabrication strongly recalls to one's mind the attitude of engineers when rolled steel plates and shapes were first put on the market. The reasons then advanced why steel was not reliable were in many respects the same as we now hear regarding welded joints. Education and experience gradually did away with all doubts, although it took some users years to overcome their prejudice. The prejudice against welding is being more rapidly dispelled, perhaps because more intensive research work has been done, and the economies of the process are more apparent and more broadly published. You may think it heresy for me to make the statement; nevertheless, I venture to say that more scientific study and research has already been done in determining the characteristics and application of welding than has been given to the actual behavior of riveted joints.

The technique of welding is well developed. Trained and reliable welders can be obtained for all kinds of operations. The application of welding in many lines of manufacture has been rapid and widespread. In structural work, the knowledge of proper details and application of welding is far behind the welding art itself. It behooves structural engineers to give careful attention to this new method, and to learn its proper and economical application. They should approach the study of welding application with open minds, and as it might be expressed, they should become "welding minded." When they reach the open-minded welding stage, many riveted details and connections will look clumsy and perhaps ineffective; and they will see how much better and cheaper they could be made by welding.

The question of cost always comes up when any new material or method is proposed. In our opinion, any figures of cost of structural welding so far done would be misleading. There are so many different operations to be readjusted in structural shop fabrication and erection, beside the welding itself, that results so far obtained can be used only as a basis for rough guesses. There is much to be learned along the line of adaptation of shop methods leading to the preparation for and the handling of material during the welding operation, and doubtless costs so far observed can be reduced. The preliminary costs do not as yet indicate that the present method of making structural joints by rivets will be entirely abandoned for economic reasons in the early future. There are many details and parts of structures that could be welded to advantage, but the all-welded bridge or building has not yet arrived. Welding can supplement riveting, each in its own place, and thereby produce the most desirable structure for the purpose, and also the most economic.

July, 1929 (2M)